**CHAPTER 4**

# INVESTIGATION OF SUBSURFACE CONDITIONS



## 4.1 INTRODUCTION

An adequate and appropriate investigation of the subsurface is critical in the design of the foundation for most projects. On occasion, information on the site can lead to a simple and straightforward approach. In parts of the world, soft soil overlies the founding stratum. Thus, the necessary investigation involves sounding to determine the thickness of the weak deposit so that the length of axially loaded piles can be selected. The situation changes, however, if the piles must sustain lateral loading, requiring detailed information on the soft soil.

For most subsurface investigations, three preliminary activities are proper: (1) gaining information on the geology at the site, as discussed in Chapter 2; (2) a field trip to the site to get specific information related to the design and construction of the foundations (this could be combined with the field trip to obtain geologic information); and (3) meetings with the architect and structural engineers to gain information on the requirements of the foundation.

Meetings with the owner and relevant professionals may lead to an early definition of the general nature of the foundation at the site, whether the foundation is shallow or deep. If the soil is soft, and if the design will require settlement and stability analyses, as for shallow foundations, Ladd (2003) presents a detailed discussion on necessary procedures for subsurface investigation and laboratory testing. If saturated clay exists at the site, the time-dependent behavior of all foundations must be considered. Soil properties are strongly influenced by the installation of deep foundations, but the prediction of the effects must start with a well-designed and effective soil investigation.

Chapter 2 discusses the desirability of a geologic investigation and indicates the availability of geologic information. Information on the geology at

the site will be valuable in planning and executing the subsurface investigation.

A field trip will determine the nature of other structures near the proposed site. If possible, information should be gained on the kinds of foundations employed in nearby buildings, any problems encountered during construction, response of the foundations since construction, and if any undesired movements have occurred. On occasion, the results will be available for nearby structures from previous subsurface investigations. Municipal agencies will provide drawings showing the location of underground lines, and power lines should be noted, along with any obstructions that would limit access by soil-boring equipment.

The condition of the site with respect to the operation of machinery for the investigation of the subsurface is important. With regard to movement of construction equipment, countries in Europe are providing guidelines for what is termed the *working platform* (European Foundations, 2004). The purpose of such guidelines is to provide a clear statement on the safety of personnel and machinery on the site. If the site is unsuitable for the operation of boring machines for soil sampling, and later for the operation of construction equipment, such guidelines will inform the owner of the site about improvements required. For example, drainage may be needed, as well as treatment of the surface of the site by an appropriate form of soil stabilization.

Meetings with other professionals on the project are essential. The tolerance of the proposed structure to movement, both vertical and lateral, should be established. The requirements of the requisite building code should be reviewed. Of most importance is the magnitude and nature of the expected loadings, whether short-term, sustained, cyclic, and/or seismic. In some instances, the probability of certain loads may be considered. The maximum loads on the foundations of many offshore structures occur during storms, whose frequency must be estimated on the basis of historical information. Discussions among the principals should address the possible effects of a foundation failure, whether a minor monetary loss, a major monetary loss, or a catastrophic failure with loss of life.

The details of the site investigation should be addressed in meetings of the principals for the project. Field and laboratory testing can be done that will have little relevance to the success of the foundation. On the other hand, evidence is clear that a thorough and proper soil investigation will affect favorably the initial and final costs of the structure. Some contracts include a clause placing responsibility on the general contractor for the subsurface investigation, possibly leading to problems as construction progresses. A better solution is to employ the phased method, noted below, and then to require the geotechnical contractor to comply with specifications tailored for the structure and site conditions.

The importance of investing appropriately in the soil investigation is illustrated in Figure 4.1. If no money is spent, the structure may collapse. Spending a small amount could lead to later expenditures to correct for unequal settlement of the foundation. As shown in Figure 4.1, an optimum amount

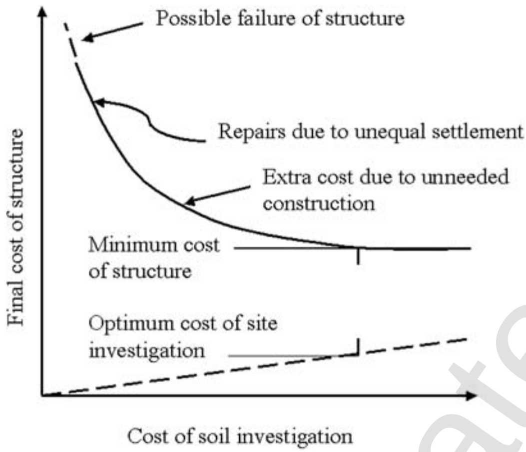



**Figure 4.1** The effect of soil investigation on the final cost of structures.

spent on the soil investigation leads to a minimum cost of the structure. If more than the optimum amount is spent, the cost of the structure increases by the cost of the soil investigation, but because the cost of the soil study normally is minor compared to the cost of the structure, the final cost of the structure increases only slightly. If owners and their representatives are aware of the facts presented in the figure, less emphasis will be put on price competition for making a soil investigation.

The ideal procedure for the subsurface investigation for a major structure is to perform exploratory borings to identify the various strata at the site and to determine whether or not the strata are tilted. The final design would then become available, and the nature of the foundation system would be evident. Borings could then be undertaken to obtain the required specimens for laboratory testing and to perform in situ tests if needed. Field loading tests could also be done if needed. This two-stage process would result in the acquisition of precise data for the design of the foundation.

The two-stage process is not possible in some instances—for example, in performing borings for the design of piles for a fixed offshore platform (see Section 4.6). Also, price competition on many projects can lead to a single-stage investigation with a limited number of borings and reduced laboratory testing. Price competition is plainly unwise when specifying a soil investigation.

## 4.2 METHODS OF ADVANCING BORINGS

### 4.2.1 Wash-Boring Technique

The use of wash borings is the most common method for advancing a boring because the technique is applicable to any soil, the depth is limited only by

the equipment employed, samples can be taken with a variety of tools, and in situ tests can be performed as the borehole is advanced. A typical drilling machine and its associated equipment are shown in Figure 4.2. The derrick is for handling the hollow drill pile that passes through a rotary table, powered by an engine with the necessary power. The hollow drill pipe carries an appropriate cutting tool. A surface casing is set with a T-section above the ground to direct the drilling water to a holding tank. A pump will drive water down the drill pile to raise the cuttings, which may be examined to gain an idea of the formation being drilled. The water is pumped from the top of the tank, with additional water provided as necessary. Drilling fluid can replace the water if caving occurs.

The system can be scaled up for drilling deeper holes and scaled down for hand operation. The Raymond Concrete Pile Company, now out of existence, distributed a movie for the classroom showing the use of a tripod, assembled on site, for raising and lowering the drill pipe by use of a pulley. A small gasoline engine rotated a capstan head used to apply tension to a rope that passed through the pulley to the drill pipe. A small gasoline pump picked up the drilling water from a tank or from an excavation on site. The entire system could be transported with a light truck and assembled and operated by two workmen. The components of such a system are shown in Figure 4.3.

### 4.2.2 Continuous-Flight Auger with Hollow Core

Borings to limited depths can be made with a continuous-flight auger driven by a powered rotary table. The stem of the auger is hollow, allowing samples to be taken through the stem without removing the auger. Boring and sampling are done rapidly but, even with a powerful rotary table, the penetration of the continuous-flight auger is limited.

**Figure 4.2** A typical drilling machine and its associated equipment.

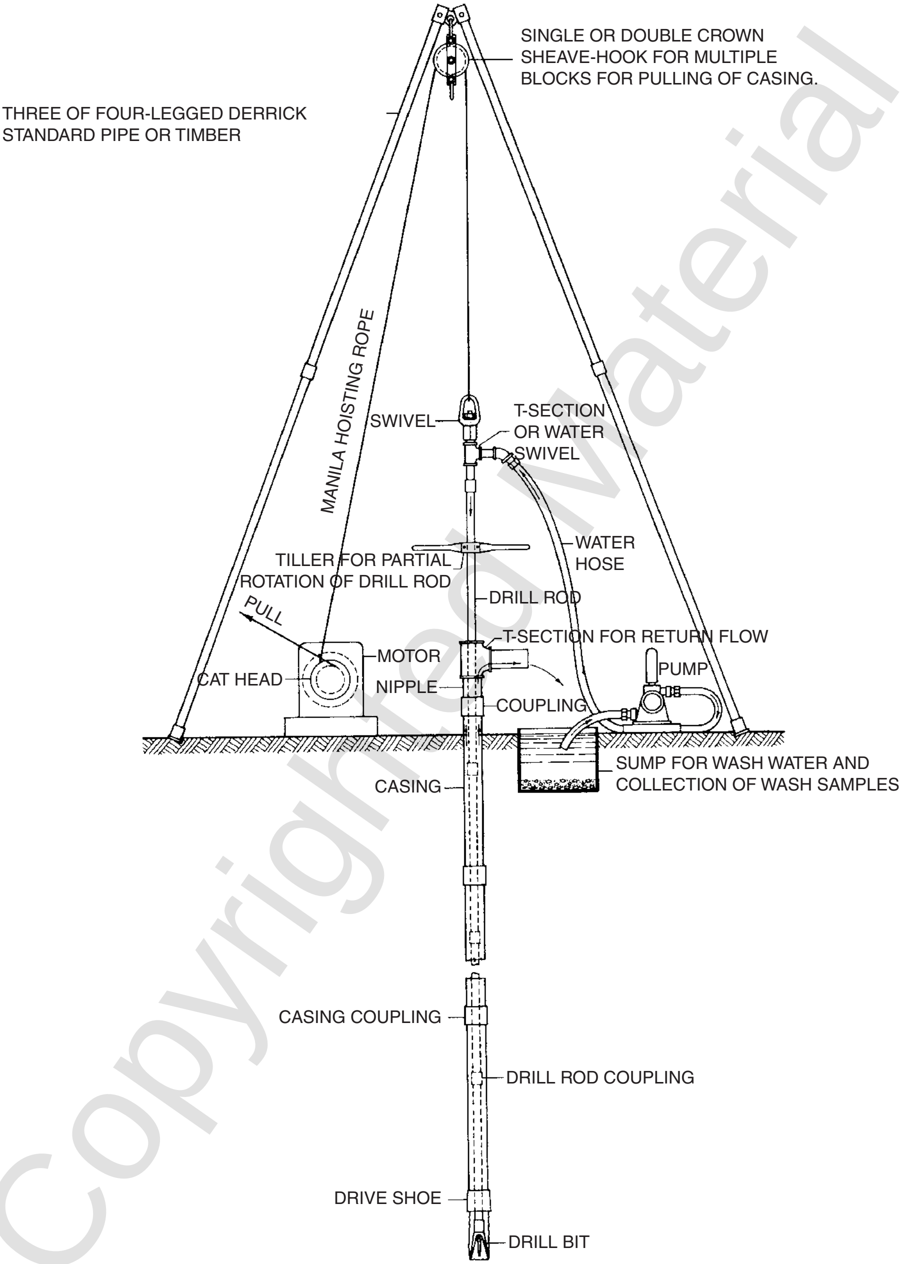



**Figure 4.3** Portable wash boring system (from Hvorslev, 1949).

The drill rod fits inside the central pipe of the auger and includes a point that is in place while the auger is advanced. When the desired depth is achieved, the drill rod with the drilling point is removed, a sampling tube is put on the drill rod, the sample is taken by pushing or driving, and the desired sample is retrieved for testing. Alternatively, an in situ testing device, as described below, can be lowered with the drill rod. After testing at a particular depth, the drill rod with the drill point can be replaced and drilling to the desired depth can be done in preparation for the acquisition of the next sample or in situ data.

## 4.3 METHODS OF SAMPLING

### 4.3.1 Introduction

In a remarkable effort, the late Dr. M. Juul Hvorslev, working at the Waterways Experiment Station in Vicksburg, Mississippi, and supported by a number of other agencies, presented a comprehensive document on subsurface exploration and sampling of soils for purposes of civil engineering (Hvorslev, 1949). The document has been reprinted by the Waterways Experiment Station and remains a valuable reference.

The term *undisturbed* is used to designate samples of high quality. The ASTM uses the term *relatively undisturbed* in describing the use of sampling with thin-walled tubes (ASTM-D 1587). Disturbance of samples is due to a number of factors: change in the state of stress as the sample is retrieved from the soil, especially if the sample contains gas; disturbance due to resistance against the sides of the sample as it enters the sampling tube; disturbance during transportation to the laboratory; and disturbance as the sample is retrieved from the sampling tube.

Disturbance due to the presence of gas in soils is difficult to overcome. Methods must be implemented to prevent the specimen from expanding throughout the sampling, transportation, trimming, and testing periods. A most severe problem involves the sampling and testing of hydrates that occur in frozen layers at offshore sites.

Sampling disturbance due to resistance against the sample as it is pushed into the sampling tube can be significant. Hvorslev (1949) includes some remarkable photographs of samples that have been distorted in the sampling process (pp. 95, 96, 98, 104, 106, 112, 115, 116). In accounting for the effects of sampling disturbance, Ladd and Foott (1974) proposed experimental procedures for use in the laboratory to determine the strength of most soft, saturated clays.

### 4.3.2 Sampling with Thin-Walled Tubes

The ASTM has published a detailed standard (1587) for thin-walled tube sampling of soils. The specified dimensions of sampling tubes are shown in Figure 4.4.

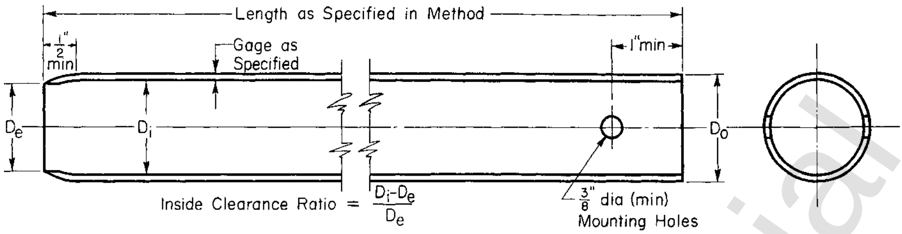



**Figure 4.4** Thin-walled tube for sampling (from ASTM D 1587).

Two mounting holes are required for 2- to 3.5-in. samplers and four mounting holes for larger sizes. Hardened screws are required for the mounting. Dimensions for tubes of three diameters are given in Table 4.1, but tubes of intermediate or larger sizes are acceptable. The lengths shown are for illustration; the proper length is to be determined by field conditions.

The preparation of the tip of the sampler is specified, and the clearance of 1 percent is designed to minimize disturbance due to resistance to penetration of the sample. The interior of the sampling tube must be clean, and a coating is sometimes recommended. Procedures for the transportation of thin-walled tubes are specified in ASTM D 4220, Standard Practice for Preserving and Transporting Soil Samples.

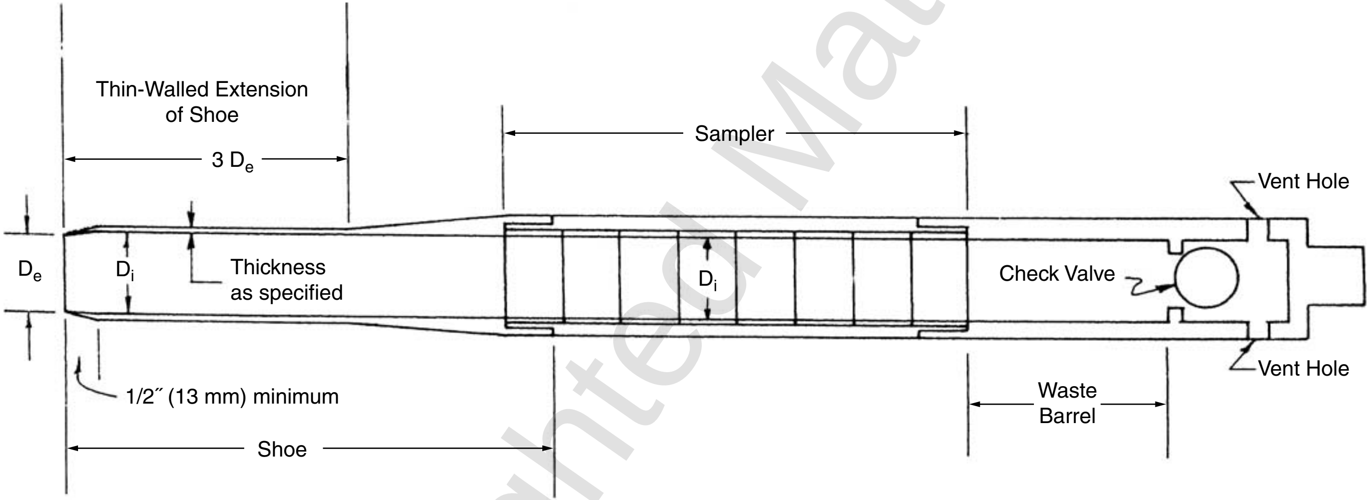
Special tools have been developed to eliminate disturbance partially due to the interior resistance of the sampling tool. At the Swedish Geotechnical Institute, a sample more than 2 m long was laid out for examination. The sample had been taken by the Swedish Foil Sampler, which consisted of a short, thin-walled section followed by a thick-walled section in which was embedded a series of rolls of foil. As the sample was pushed into full penetration, the rolls of foil were simultaneously pulled back to eliminate completely resistance due to sample penetration. The description of a similar device is presented in ASTM D 3550, Standard Practice for Ring-Lined Barrel Sampling of Soils. A sketch of the sampling tool is shown in Figure 4.5.

The ASTM Book of Standards presents 13 standards, in addition to D1587, related to surface and subsurface characterization of soil and rock. A list of the relevant ASTM standards is presented in Appendix 4.1.

**TABLE 4.1 Suitable Thin-Walled Steel Sampling Tubes**

| | | | |
|---|---|---|---|
| Outside diameter, in. | 2 | 3 | 5 |
| Wall thickness, in. | 0.049 | 0.065 | 0.120 |
| Tube length, in. | 36 | 36 | 54 |
| Clearance ratio, % | 1 | 1 | 1 |

*Source:* ASTM D 1587-83




NOTE 1—Inside clearance ratio = $(D_i - D_e)/D_e$
NOTE 2—Dimensional tolerance of $D_i$ = ±0.003 in. (±0.08 mm)

**Figure 4.5** Ring-lined barrel sampling assembly (from ASTM D 3550).

### 4.3.3 Sampling with Thick-Walled Tubes

Exploratory investigations can be made using a thick-walled sampler that has a split barrel. The sampler can be opened in the field and the contents examined in order to log the stratum being bored. A sketch of the sampler is shown in Figure 4.6, taken from ASTM Standard D 1586, Standard Method for Penetration Test and Split-Barrel Sampling of Soils. After the boring has penetrated to the desired depth, the sampler is fixed to sampling rods and the Standard Penetration Test (SPT) can be performed.

The SPT is performed by dropping a 140-lb weight a distance of 30 in. to impact the top of the sampling rods. The blows required to drive the sampler for each of three 6-in. intervals are counted. The *N*-value is the sum of the number of blows required to drive the sampler through the second and third intervals. The SPT has been used for many years as an exploratory tool and sometimes to gain information for design. Undisturbed samples of sand cannot be taken except sometimes in the capillary zone or by freezing. As noted in Chapter 3, correlations have been proposed for values of the friction angle $\phi$ as a function of *N*. Even though the sample of sand is disturbed, it can be examined for grain shape and character, and grain-size distribution curves can be developed.

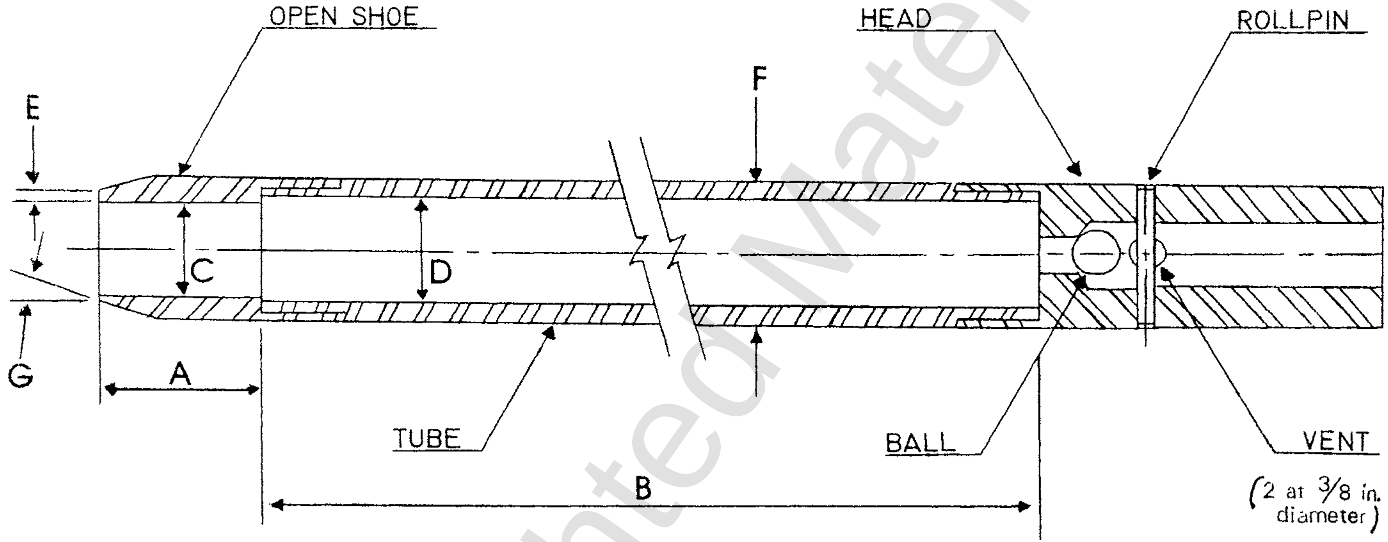
Samples of clay obtained by the split-spoon sampler can be examined in the field to gain information on the character of the deposit. Specimens can be taken to the laboratory for determination of water content and for determining Atterberg limits. However, correlations between shear strength and *N*-value for clay soils are not recommended. Most clays are either saturated or partially saturated, and the porewater pressures in the clay below the impacted sampling tool are certainly affected. Clays can be sampled with the thin-walled tube and tested in the laboratory for strength and other characteristics.

Soil sampling methods are not applicable if the blow count reaches 50 for a penetration of 1 inch. The material can then be sampled by core drilling.

### 4.3.4 Sampling Rock

Samples of rock can be taken by core drilling, as shown in Figure 4.7, and the standard practice for diamond core drilling for site investigation is given in ASTM D 2113, as shown in Figure 4.6. The *rock quality designation* (RQD) should be recorded, and is defined in percentage terms as determined by summing the length of the sound pieces of core that are at least 4 in. long and dividing that length by the length of core drilled. The RQD will vary, of course, through the thickness of a stratum, and the RQD should be recorded for a specified length of core.

Intact specimens of core can be tested in a triaxial device (ASTM D 2664) or in unconfined compression (ASTM D 2938). Tensile strength and elastic moduli may be tested using intact cores if the values are needed in design.




A = 1.0 to 2.0 in. (25 to 50 mm)
B = 18.0 to 30.0 in. (0.457 to 0.762 m)
C = 1.375 ± 0.005 in. (34.93 ± 0.13 mm)
D = 1.50 ± 0.05 − 0.00 in. (38.1 ± 1.3 − 0.0 mm)
E = 0.10 ± 0.02 in. (2.54 ± 0.25 mm)
F = 2.00 ± 0.05 − 0.00 in. (50.8 ± 1.3 − 0.0 mm)
G = 16.0° to 23.0°

The 1½ in. (38 mm) inside diameter split barrel may be used with a 16-gage wall thickness split liner. The penetrating end of the drive shoe may be slightly rounded. Metal or plastic retainers may be used to retain soil samples.

**Figure 4.6** Split-barrel sampler (from ASTM D 1586).

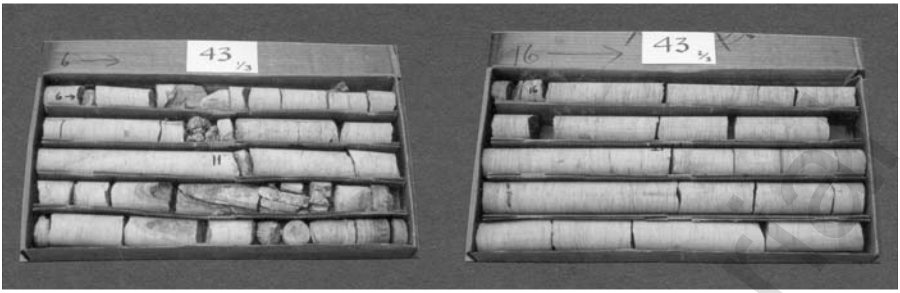



**Figure 4.7** Rock-core samples.

During the investigation of the site, the face of the rock for the foundation may be exposed. If so, a comprehensive examination of the face of the rock is recommended (Gaich et al., 2003; Lemy and Hadjigeorgiou, 2003).

## 4.4 IN SITU TESTING OF SOIL

### 4.4.1 Cone Penetrometer and Piezometer-Cone Penetrometer

Several types of cone penetrometers are in use, including the mechanical cone, mechanical-friction cone, electric cone, electric-friction cone, and piezometer cone. The mechanical cone has a 60° point angle and a 1.406-in. base diameter. The cone is attached to hollow drill rods and may be pushed down about 3 in. by push rods inside the hollow drill rods while measuring the push force. The mechanical-friction cone has a point of the same size as the mechanical cone and, in addition, includes a sleeve that will be engaged and pulled down after the cone has been pushed down. Two resistances are measured, the resistance from the cone only and the resistance from the cone and the friction sleeve. Sketches of the friction-cone penetrometer are shown in Figure 4.8.

The electric-cone penetrometer has a cone the same size as the mechanical cone, but the resistance to penetration is measured by a load cell at the top of the penetrometer. Readings are taken by an electrical conduit coming to the ground surface. The electric-friction-cone penetrometer is similar to the electric-cone penetrometer, but two measurements on resistance are taken as the device is pushed into the soil. Load cells from strain gauges allow the cone resistance and the friction-sleeve resistance to be measured simultaneously. Sketches of the electric-cone penetrometer and the electric friction-cone penetrometer are shown in Figures 4.9 and 4.10, respectively.

The piezometer-cone penetrometer combines the electric cone with a piezometer that can be read electronically. In addition to yielding the cone resistance, the device gives information on the value of the porewater pressure at the depth of the test. The additional information gives the engineer a way to predict more accurately the characteristics of the stratum being investigated.

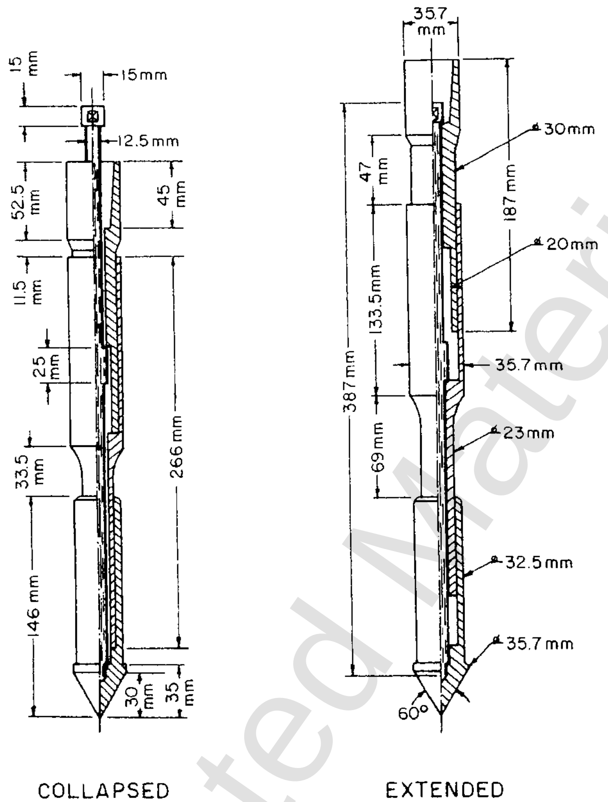



**Figure 4.8** Example of a mechanical friction-cone penetrometer tip (Begemann friction-cone) (from ASTM D 3441).

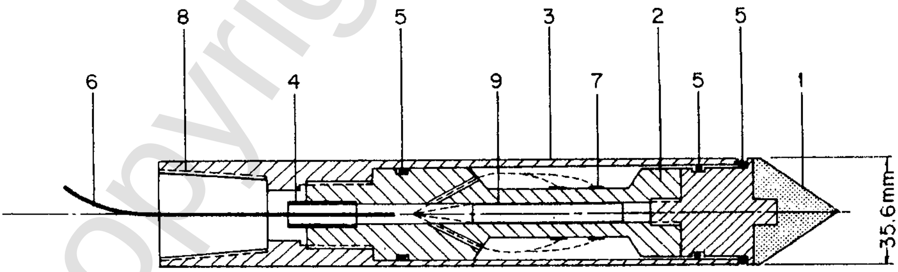



**Figure 4.9** Electric-cone penetrometer tip (from ASTM D 3441).

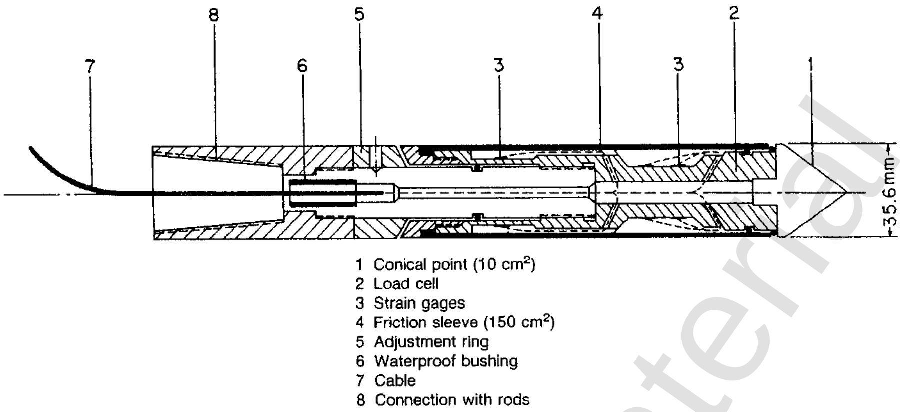



**Figure 4.10** Electric friction-cone penetrometer tip (from ASTM D 3441).

ASTM provides the following statement about the quality of the data collected by the cone penetrometer (ASTM D 3441, p. 475): "Because of the many variables involved and the lack of a superior standard, engineers have no direct data to determine the bias of this method. Judging from its observed reproducibility in approximately uniform soil deposits, plus the $q_c$ and $f_c$ measurement effects of special equipment and operator care, persons familiar with this method estimate its precision as follows: *mechanical tips*—standard deviation of 10% in $q_c$ and 20% in $f_c$; *electric* tips—standard deviation of 5% in $q_c$ and 10% in $f_c$." If the shear strength of clay is to be determined, the engineer must divide the value of $q_c$ by a bearing-capacity factor; opinions vary about what value of that factor should be employed.

### 4.4.2 Vane Shear Device

Field vane testing consists of inserting vanes at the ends of rods into soft, saturated soils at the bottom of a borehole and rotating the rods to find the torsion that causes the surface enclosing the vane to be sheared. The torsion is converted into a unit shearing resistance. Two views of typical vanes are shown in Figure 4.11. If the rod used to insert the vane is in contact with the soil, a correction must be made for the torsion on the rod.

With the vane in position, the first test is performed by rotating the rod attached to the vane at a rate not exceeding 0.1° per second, usually requiring 2 to 5 minutes to achieve the maximum torque, yielding the undisturbed shear strength. Then the vane is rotated rapidly through a minimum of 10 revolutions to remold the soil. Finally, the test is repeated to obtain the remolded shear strength of the soil.

The shear strength, $s$ (lbf/ft$^2$), is found from the following equation:

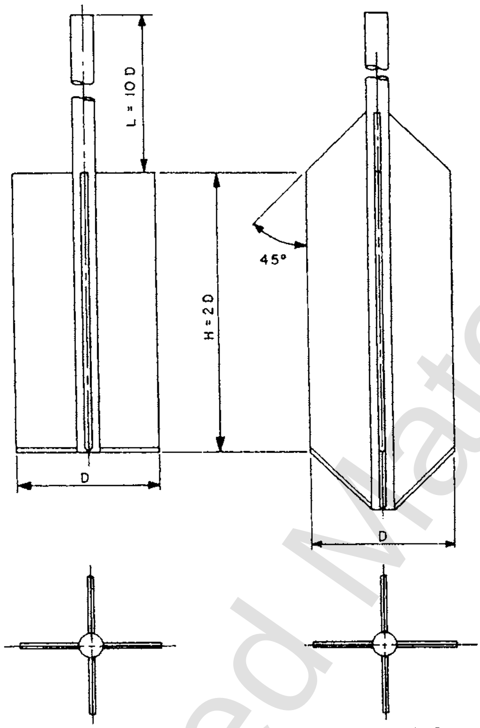



**Figure 4.11** Geometry of a field vane (from ASTM D 2573).

$$s = \frac{T}{K} \tag{4.1}$$

where $T$ is torque in lbf ft and $K$ is a constant in ft$^3$, depending on the shape of the vane. As an example, the value of $K$ for the rectangular vane shown in Figure 4.10 is as follows:

$$K = \left(\frac{\pi}{1{,}728}\right)\left(\frac{D^2H}{2}\right)\left(1 + \left(\frac{D}{3H}\right)\right) \tag{4.2}$$

where

$D$ = measured diameter of the vane, in., and
$H$ measured height of the vane, in.

A problem with no easy solution is one in which the soil has inclusions such as shells. The value of *s* from the vane would be higher than the actual shear strength and could lead to an unsafe design.

### 4.4.3 Pressuremeter

The pressuremeter test consists of placing an inflatable cylindrical probe into a predrilled hole and expanding the probe in increments of volume or pressure. A curve is obtained showing the reading of the volume as a function of the pressure at the wall of the borehole. Baguelin et al., (1978) state that Louis Ménard was the driving force behind the development of the pressuremeter. Early tests were carried out in 1955 with a pressuremeter designed by Ménard, a graduate student at the University of Illinois. Ménard later established a firm in France where the pressuremeter was used extensively in design.

The components of the pressuremeter system are shown in Figure 4.12. The probe consists of a measuring cell and two guard cells. The cells are filled with water, and gas is used to expand the cells against the wall of the borehole. The guard cells are subjected independently to the same pressure as the measuring cell, and all three cells are expanded at the same rate, ensuring two-dimensional behavior of the measuring cell. A borehole is dug slightly larger in diameter than the diameter of the pressuremeter probe, with the nature of the soil being noted at the depths of the tests. The test is per-

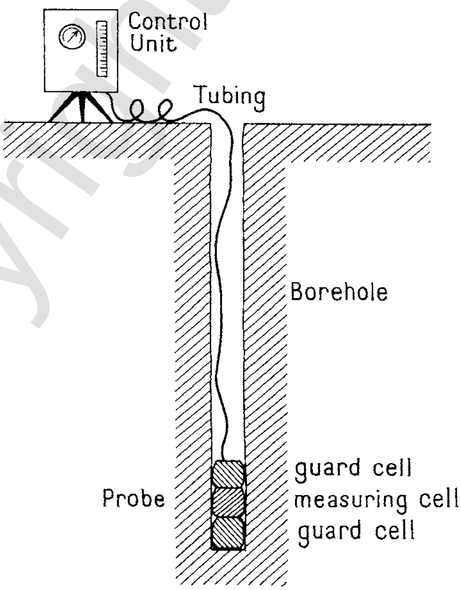



**Figure 4.12** Components of the pressuremeter system (from Baguelin et al., 1978).

formed according to a standard procedure—for example, according to ASTM D 4719.

The results from a pressuremeter test are presented in Figure 4.13. The following data apply with respect to the curve: type of soil, silty clay; depth of test, 7 m, and depth to water table, 1.5 m. The following values are from the curve. The volume at the beginning of the straight-line portion of the curve, $v_0 = 170\ \text{cm}^3$; the volume at the end of the straight-line portion of the curve, $v_f = 207\ \text{cm}^3$; and the limit pressure, $p_l = 940$ kPa. The volume of the pressuremeter when the pressure was zero was 535 cm$^3$.

The pressuremeter modulus may be computed from the following equation (ASTM D 4719). Values from Figure 4.13 are shown where appropriate.

$$E_p = 2(1 + v)(V_0 + V_m)\frac{\Delta P}{\Delta V} \tag{4.3}$$

where

$E_p$ = an arbitrary modulus of deformation as related to the pressuremeter, kPa,
$v$ = Poisson's ratio, taken as 0.33,
$V_0$ = volume of the measuring portion of the probe at zero reading of the pressure (535 cm$^3$),
$V_m$ = corrected volume reading at the center of the straight-line portion of the pressuremeter curve, measured as (170 + 207)/2), and
$\Delta P/\Delta V$ = slope of the straight-line portion of the pressuremeter curve, (measured as 13.7-kPa/cm$^3$).

$$E_p = 2(1 + 0.33)(535 + 188.5)(13.7) = 26{,}000\ \text{kPa}$$

Some error is inherent in the value of $E_p$ due to the measurements and the value is an arbitrary measure of the stiffness of the soil, but the engineer can gain some useful information. The most useful information is presented by Baguelin et al. (1968), where correlations are given for the net limit pressure and properties of clay and sand (see Tables 4.2 and 4.3.)

The net limit pressure, $p_1^*$, is equal to the limit pressure minus the total initial horizontal pressure where the pressuremeter test was performed, as found from the following equation:

$$p_1^* = p_1 - [(z\gamma - u)\ K_0 + u] \tag{4.4}$$

where

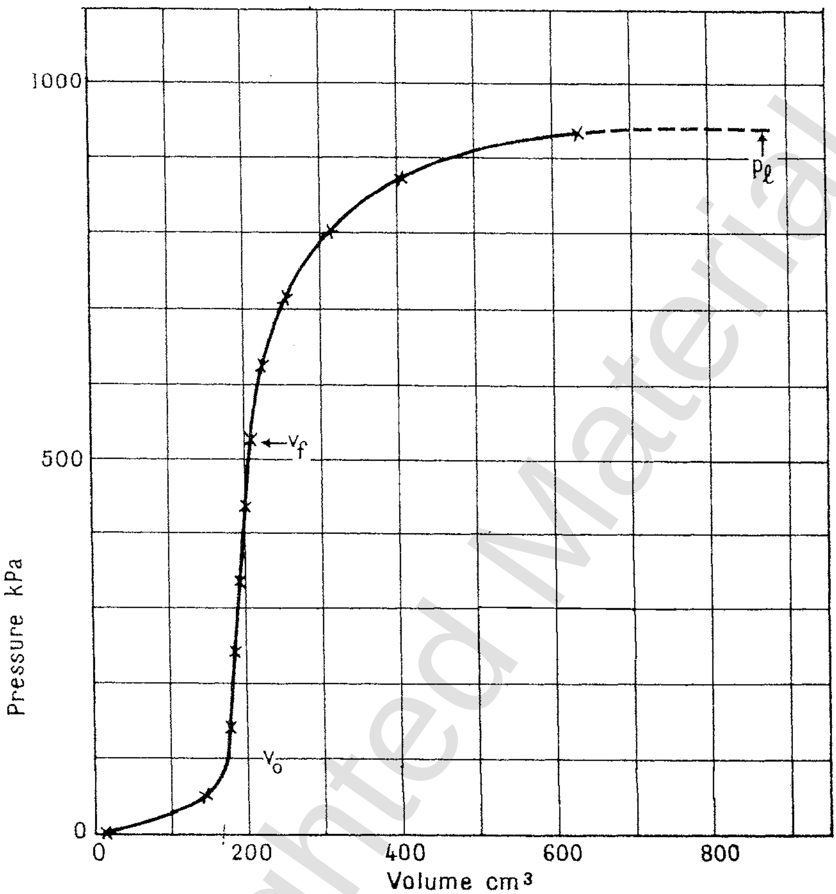



**Figure 4.13** Corrected pressuremeter curve (from Baguelin et al., 1978).

$z$ = depth below ground surface where test was performed,
$\gamma$ = unit weight of soil,
$u$ = porewater pressure, and
$K_0$ = coefficient of earth pressure at rest.

Use of the pressuremeter that is installed in a predrilled hole results in a pressure that is zero, or lower than the earth pressure at rest if the excavation is filled with water or drilling mud; thus, some creep of the soil could occur inside the borehole. In some instances, the test cannot be performed if the borehole collapses. A self-boring pressuremeter has been developed; it can be installed with a minimum of disturbance of the soil. The advantage of the self-boring pressuremeter is obvious; the disadvantages are that extra time is required to perform the test and, in some instances, the pressuremeter cannot be recovered.

**TABLE 4.2 Correlations between Properties of Clay and Limit Pressure from the Pressuremeter Test**

| $p_l^*$, kPa | Description | Field Test | $p_l^*$, psi |
|---|---|---|---|
| 0–75 | Very soft | Penetrated by fist; squeezes easily between fingers | 0–10 |
| 75–150 | Soft | Penetrated easily by finger; easily molded | 10–20 |
| 150–350 | Firm | Penetrated with difficulty; molded by strong finger pressure | 20–50 |
| 350–800 | Stiff | Indented by strong finger pressure | 50–110 |
| 800–1600 | Very stiff | Indented only slightly by strong finger pressure | 110–230 |
| 1600+ | Hard | Cannot be indented by finger pressure; penetrated by fingernail or pencil point | 230+ |

*Source:* Baguelin et al. (1968).

**TABLE 4.3 Correlations between Properties of Sand and Limit Pressure from the Pressuremeter Test**

| $p_1^*$, kPa | Description | SPT $N$ | $p_1^*$, psi |
|---|---|---|---|
| 0–200 | Very loose | 0–4 | 0–30 |
| 200–500 | Loose | 4–10 | 30–75 |
| 500–1000 | Compact | 10–30 | 75–220 |
| 1500–2500 | Dense | 30–50 | 220–360 |
| 2500+ | Very dense | 50+ | 360+ |

*Source:* Baguelin et al. (1968).

## 4.5 BORING REPORT

The quality of a boring report is related to a number of critical features. Most important, a coordinate system must be established before the boring begins. Preferably, the coordinate system should be the one employed by the local governmental entity, but a site-specific system can be established if necessary. Horizontal and vertical controls are necessary. The horizontal coordinates and the elevation of each boring must be shown.

On occasion, the information from the field is analyzed and the boring report is combined with data from the analyses. There are strong reasons to believe that the information from the field should be presented in detail. Information of importance is the weather, personnel on the job and their responsibilities, time needed for each operation, equipment used to advance the boring, measures used to keep the borehole open if necessary, kinds of samples taken, any difficulty in sampling, depths of samples, in situ tests performed and depths, and description of the soils removed.

An engineer-in-training or a registered engineer should be on the job to log the information on the soils recovered from each borehole. As much as possible, the nature of the soils encountered and the grain sizes should be noted. Some field tests may be possible to ascertain the strength of the samples recovered. A hand-held penetrometer can be used to obtain and estimate the strength of clays, and in some cases the use of a miniature vane at the ends of a sample of clay in the sampling tube can reveal in situ shear strength. The engineer on site will have knowledge of the structure to be placed on the site and can acquire relevant information that may be difficult to list at the outset of the work. For example, the engineer will know if the structure is to be subjected to lateral loads and that the character of the soils near the final ground surface is quite important in performing analyses to determine the response of foundations. Special procedures may be necessary to find the strength and stiffness of the near-surface soils.

An open borehole should remain on the site of the exploration for the location of the water table, information that is critical in the design of foundations. Frequently, water or drilling mud is used in drilling the borehole, so

drilling fluid must be dissipated in order to get the depth to the water table. If the water table is in granular soil, sounding of the borehole with a tape will reveal its location with little delay. However, if the water table is in fine-grained soil, the borehole must be left open for some time, requiring a workman to return to the site, perhaps on several occasions. After the water in the borehole achieves the same level for some time, the borehole may be filled with grout if required by the specifications.

The engineer must decide how deeply to drill the boreholes for the subsurface investigation. The problem has a ready solution if a founding stratum, such as bedrock, with substantial thickness exists at a reasonable depth. If no preliminary design has been made, the engineer charged with the subsurface investigation must decide on the probable type of foundation based on available information. For shallow foundations, computations should be made showing the distribution of pressure with depth. The borings should be extended to the depth below which the reduced pressure will cause no meaningful settlement. For deep foundations, computations should be made for the distribution of pressure below the tips of the piles to check for undesirable settlement. A problem would occur if the piles are designed as end bearing in a relatively thin stratum above compressible soil.

## 4.6 SUBSURFACE INVESTIGATIONS FOR OFFSHORE STRUCTURES

The initial step in an offshore investigation is to refer to a detailed map of the topography of the sea bed. Such a survey will be available from the owner of the site. In addition to the location of the structure to be constructed, the topography of the sea bed in the general area should be studied. There may be evidence of the existence of stream beds that have been refilled with soft material. A further step would be to review geophysical tests that have been performed at the site. In the North Sea, it is customary to use high-power sparker profiling with a spacing of 200 to 1000 m (Andresen et al., 1978).

An investigation of the character of the soils at an offshore site usually requires a special vessel, such as that shown in Figure 4.14. The daily cost for the drilling vessel is substantial, so double crews are necessary to allow the subsurface investigation to continue around the clock. A weather window is necessary to allow the vessel to stay on site for the few to several days required to complete the work on site.

Over the several decades that offshore borings have been made, procedures have been developed to allow the work to proceed with dispatch. Each rig includes a mechanism, as shown in Figure 4.14, to compensate for the heave of the ship. The ship goes to the drill site and is anchored in multiple directions; a casing is then placed on the ocean floor, and soil exploration proceeds. Drilling is accomplished through the drill string, and drilling fluid may be used to maintain the excavation if necessary. The common procedure is to lower a sampling tool with a wire line; the wire line can raise a weight, and

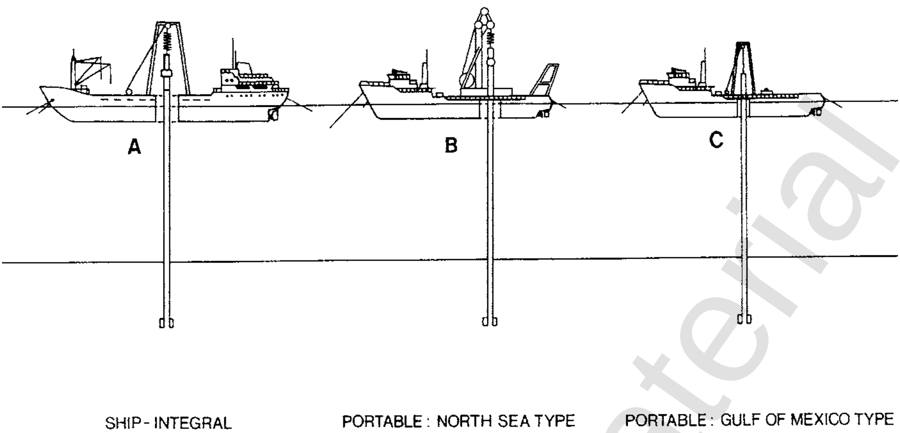



**Figure 4.14** Sketches of different types of ships and drilling systems (from Richards and Zuidberg, 1983).

the tool is driven into position. The end of the drill string and the sampling tool are shown on the left-hand side of Figure 4.15 (Richards and Zuidberg, 1983). The sample is retrieved by lifting the wire line, and the borehole is advanced by drilling to the next position for sampling.

An alternative method of sampling is shown on the right-hand side of Figure 4.15. The wire line lowers a hydraulic piston and sampling tube. The tool can be activated from the drill ship, and the thin-walled sampler is pushed into place. A number of other tools may be lowered; however, the hammer sampler is the most common because of the extra work in latching more complex tools to the bottom of the drill string.

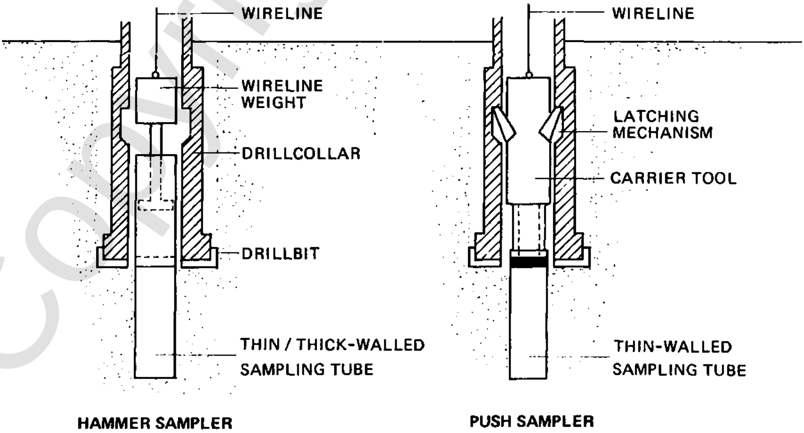



**Figure 4.15** Diagrams of two types of wire line samplers (from Richards and Zuidberg, 1983).

Emrich (1971) reported on a series of studies aimed at comparing the results of the properties of clays obtained by various techniques in borings done to a depth of 91 m at an offshore site near the Mississippi River. Samples were taken using a hammer sampler with a diameter of 57 mm, with a hammer sampler with a diameter of 76 mm, with an open-push sampler with a diameter of 76 mm, and with a fixed-piston sampler with a diameter of 76 mm. Unconfined compression tests were performed on samples from each of the three methods of sampling. In addition, field vane tests were performed, and miniature vane tests were performed at the ends of some of the samples within the sampling tubes.

The data were analyzed by plotting the results on the same graph, with the results from the fixed-piston sampler being taken as the correct value. Compared to results from the fixed-piston sampler, the following results were obtained: 57-mm hammer sampler, 64%; 76-mm hammer sampler, 71%; open-push sampler, 95%. The results from the vane tests were scattered but were generally higher than those from the fixed-piston sampler. Sampling disturbance can explain the lower values of shear strength for some of the methods described above. To reduce the effect of sampling disturbance, some investigations subjected the specimens from the hammer sampler in a triaxial apparatus to a confining pressure equal to the overburden pressure. This procedure could be unwise if the soil at the offshore site is underconsolidated, as are many recent offshore deposits.

Many offshore investigations are performed at a site where the loads of the piles have been computed and the borings are to be carried deep enough so that information is available on the required penetration of the piles, employing the guidelines of the appropriate governing authority, such as the American Petroleum Institute (API, 1987). Because offshore platforms are frequently designed to sustain horizontal loads during a severe storm, some piles will be subjected to large tensile loads. The exploration must alert the construction managers if soils at the site may be impossible to penetrate, even with a very-heavy-impact hammer, so that proper procedures can be in place to allow construction to proceed without undue delay.

Factors involving soil properties that are critical to the design of piles under axial loading are numerous. The reader is referred to Chapters 10, 11, 12, 13, and 14 for further reading.

## PROBLEMS

**P4.1.** Prepare a list of eight items you would investigate during a visit to the site of a new project.

**P4.2.** A meeting has been scheduled with the representatives of the owner of the project, the architect, and the structural engineer. List the questions you would ask to obtain needed information prior to going to the site and initiating the soil investigation.

**P4.3.** Soil investigations in the past have shown that the soil at the site of the project consists of soft clay over a founding stratum of dense sand. The decision has been made to use axially loaded piles with end bearing in the sand. Describe the techniques you would employ to obtain the thickness of the clay layer and the relative density of the sand.

**P4.4.** You are the owner of an established geotechnical firm with experience and are to be interviewed by representatives of the owner of a site where a high-rise building is to be constructed about performing a subsurface investigation at the site. (a) List the points you would make in stating that a preliminary investigation should be funded before reaching an agreement for the comprehensive investigation. (b) You need the work; if the owner insisted, would you give a fixed price for the comprehensive investigation?

**P4.5.** Discuss whether or not, as an owner of a geotechnical firm, you would want an engineer at the site of a subsurface investigation or would be comfortable having an experienced technician in charge.

**P4.6.** List the in situ methods mentioned in the text in the order of their complexity, with the least complex method first.

## APPENDIX 4.1

| 1992 Annual Book of ASTM Standards, Volume 04.08 Soil and Rock, Surface and Subsurface Characterization, Sampling and Related Field Testing for Soil Investigation | |
|---|---|
| D 420-87 | Guide for Investigating and Sampling Soil and Rock |
| D 1452-80 (1990 | Practice for Soil Investigation and Sampling by Auger Borings |
| D 1586-84 | Method for Penetration Test and Split-Barrel Sampling of Soils |
| D 1587-83 | Practice for Thin-Walled Tube Sampling of Soils |
| D 2113-83 (1987) | Practice for Diamond Core Drilling for Site Investigation |
| D 2573-72 (1978) | Test Method for Vane Shear Test in Cohesive Soil |
| D 2664 | Standard Test Method for Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements |
| D 2936 | Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens |
| D 2938 | Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens |
| D 3148 | Standard Test Method for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression |
| D 3441-86 | Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soils |
| D 3550-84 (1991) | Practice for Ring-Lined Barrel Testing of Soils |
| D 3967 | Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens |
| D 4220-89 | Practice for Preserving and Transporting Soil Samples |
| D 4428/D 4428M–91 | Test Method for Crosshole Seismic Testing |
| D 4633-86 | Test Method for Stress Wave Energy Measurement for Dynamic Penetrometer Testing Systems |
| D 4719-87 | Test Method for Pressuremeter Testing of Soils |
| D 4750-87 | Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well) |
| D 5195-91 | Test Method for Density of Soil and Rock In-Place at Depths Below the Surface by Nuclear Methods |